1st Za Chieh-Moh Distinguished Lecture: The Geotechnical Problems of The Second World Largest Copper Tailings Pond at Zelazny Most, Poland

M. Jamiolkowski[1], W.D. Carrier[2], R.J. Chandler[3], K. Hoeg[4], W. Swierczynski[6] and W. Wolski[5]
[1]*Emeritus Professor, Technical University of Torino, Italy, E-mail* michele.jamiolkowski@polito.it;
[2]*Ph.D., Consultant, Lakeland, Fla, USA, E-mail* wdcarrier@verizon.net;
[3]*Emeritus Professor of Geotechnical Engineering, Imperial College, London, UK, E-mail* r_chandler@tiscali.co.uk;
[4]*Professor, Norwegian Geotechnical Institute (NGI) and University of Oslo, Norway, E-mail* kh@ngi.no;
[5]*Professor, Geoteko Ltd, Warsaw, Poland, E-mail* info@geoteko.com.pl;
[6]*M.Sc. KGHM, Lubin, Poland, E-mail* wieslaw.swierczynski@kghm.pl.

ABSTRACT: The paper summarizes the experiences of the writers in assisting KGHM (the Polish acronym for Copper Mine and Mill Company) in the development of one the world's largest copper tailings disposal facility, located in South-West Poland. This paper describes the tailings disposal facilities, the geological features of the area with particular emphasis on the considerable influence of the various Pleistocene glaciations, and the geotechnical aspects of the design and construction. The geotechnical characterization of the tailings and of the foundation soils is described, focusing on their shear strength, on the mining-induced seismicity and on the factors controlling the stability of the ring-dam that confines the tailings. Finally, the results of the analyses of the stability of the dam, together with the details of the intensive monitoring of the performance of the dam, the latter carried out by KGHM, are presented.

1. INTRODUCTION

The paper illustrates the geotechnical aspects involved in the development of one of the world's largest copper tailings disposal at Zelazny Most in South-west Poland, close to the borders with the Czech Republic and Germany, Fig. 1.

Figure 1. Zelazny Most copper tailings disposal location in Poland

KGHM, a government owned mining company, started extracting copper ore in 1972, planning to continue until exhaustion of the ore body, estimated to occur in 2042. The ore-body is exploited by means of three different mines. At present, more than 480 Mm3 of tailings are stored in the disposal area, and the maximum height of the dam is close to 60 m. An aerial view of the Zelazny Most disposal is shown in Fig. 2.

Figure 2. Tailings disposal, aerial view

As with all tailings dams, Zelazny Most poses a number challenges to the geotechnical engineers involved in their design and construction, and particularly the recurring subject of the possibility of flow liquefaction of the stored tailings, a phenomenon which has been responsible for the collapse of several tailings dams, and which frequently involve casualties. Another issue of concern for the designers of tailings dams is the stability of the dams, which depends on the height of the dam and the mechanical behaviour of the foundation soils.

The Zelazny Most tailings disposal is situated in an area of Poland that during the Pleistocene experienced at least three glaciations, see Fig. 3. The ice sheets that over-rode the Zelazny Most area induced substantial glacio-tectonic phenomena, the most important of which was the formation of extensive, sub-planar shear zones of remarkable depths in the underlying high plasticity Pliocene clay. Where these shear surfaces are present the available shear strength is close to residual, and consequently they pose a particular challenge to the geotechnical engineer responsible for the stability of the dams. In consideration of the magnitude and extent of the Zelazny Most scheme (the ring-dam is almost 15 km in circumference), and its importance for the continuing operation of the mining operations, KGHM appointed a four-member International Board of Experts (IBE). The IBE was formed in 1992, with the task, in cooperation with the Polish geotechnical expert (PGE) Prof. W. Wolski, of overseeing the safe development of the tailings dams by applying the observational method (Peck, 1969; 1980).

Figure 3. Maximum extension of the Pleistocene glaciations in South-West Poland

This paper, co-authored by the IBE members, by the PGE and by a KGHM representative, summarizes the work carried out over the last 18 years to improve the geotechnical characterisation, to enhance the monitoring system, and to gain a deeper insight into the geotechnical problems related to the safe development of the Zelazny Most tailings disposal.

2. TAILINGS DISPOSAL

Each year, the international mining industry processes hundreds of millions of tonnes of earth and rock to extract the industrial, construction, and energy minerals that are the foundation of our modern technological civilization. A large portion of ore is waste mineral material, commonly referred to as tailings. In some cases, such as copper, the tailings often constitute more than 99% of the original ore. In most cases, the tailings are transported hydraulically from the mine beneficiation plant to a disposal area, referred to as a tailings pond. There the tailings are deposited; the water is decanted and re-cycled back to the beneficiation plant. The tailings pond includes a dam to retain the tailings; depending on the local topography, the dam may simply cross a valley, or it may form a dike around the entire pond. Tailings ponds are probably the largest man-made structures on Earth (ICOLD 2001). Typically, they are constructed over a period of decades. There are three basic types of tailings dams: downstream, upstream, and centreline (Vick 1983; Carrier 2003). Downstream construction is used when there is a sufficient volume of the coarse fraction of the tailings to construct the entire dam. A downstream dam is raised in a series of lifts as the level of the tailings rises during the course of mining and processing. Because the centreline of the crest of the dam and the downstream slope and toe move downstream as each new lift is added, the topography and property boundaries must also be appropriate. In certain cases, none of the tailings is suitable for raising a dam. Instead, local earthen materials are utilized, similar to conventional water supply dams. These are also often referred to as downstream tailings dams, in order to distinguish them from upstream tailings dams.

Upstream construction is used when the tailings are suitable for raising a dam. The coarse fraction of the tailings (i.e. sand) is separated from the fine fraction (i.e. silt and clay, usually called “slimes”) by means of either spigots or cyclones that are periodically moved along the crest of the embankment as it is raised. A slurry of fine material runs down the beach into the pond; the coarse material is used to construct the shell in a series of lifts. The centreline of the crest moves upstream as each new lift is added. Centreline construction is a hybrid of the downstream and upstream methods. Coarse shell material is added in lifts, both upstream and downstream, so that the centreline of the crest of the dam remains in the same location throughout the life of the structure.

The Zelazny Most tailings dam is being raised by the upstream method, Fig. 4. It receives approximately 80,000 tonnes/day of tailings from the three underground copper mines. The tailings from the Lubin and Rudna Mines are siliceous and coarser, and are used to construct the shell of the dam by means of spigotting; the tailings from the Polkowice Mine are carbonaceous and finer and are deposited directly into the pond.

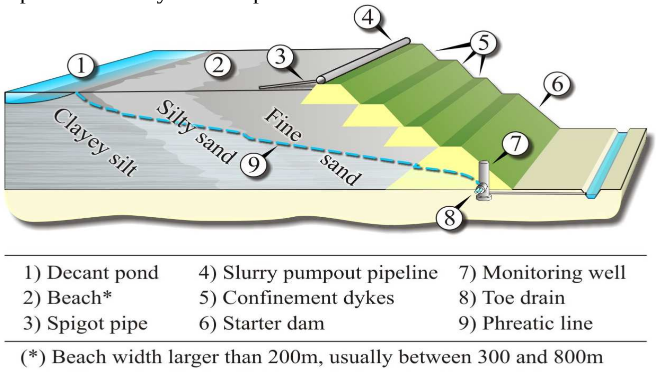


Figure 4. Schematic cross-section of Zelazny Most tailings disposal

The Zelazny Most upstream tailings dam completely surrounds the pond; in plan view, it looks roughly like a motorcycle helmet, Fig. 5. The original ground surface was saddle-shaped, such that the eastern and western portions of the dam are higher than the Northern and Southern portions. A small stream, the Kalinówka River, had its headwaters in the central area of the Zelazny Most pond, and flowed eastward. Presently, the crest of the dam is at approximately elevation 170 m above sea level (asl). The height of the dam above the downstream natural grade varies from approximately 22 to 60 m. Zelazny Most is the largest tailings dam in Europe, and it is among the largest upstream tailings dams in the world.

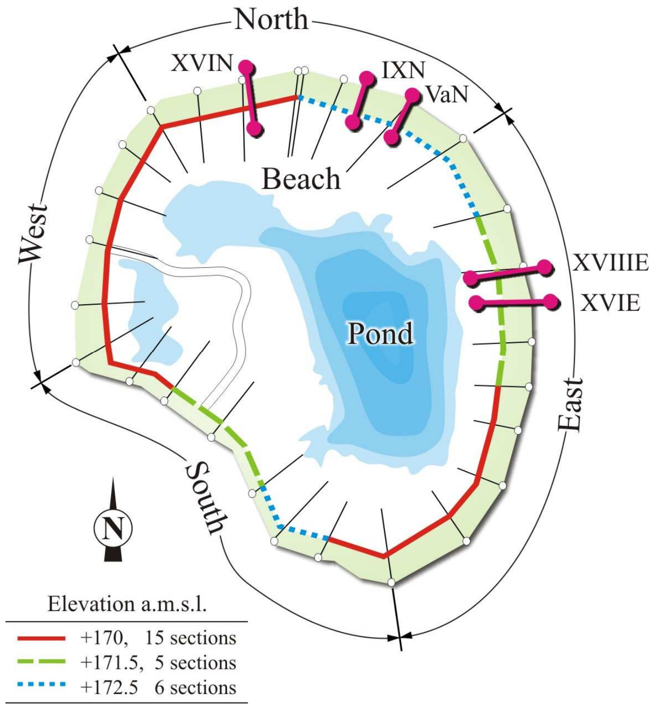


Figure 5. Dams elevations and relevant cross-sections

Construction of the Zelazny Most starter dam from local earthen materials began in 1975; and deposition of tailings began in 1977.

Since then, the crest of the tailings has risen at a rate of approximately 1.25 to 1.5 m/yr, with a downstream slope of approximately 3.5 horizontal to 1 vertical. Furthermore, the beach (the distance from the crest to the edge of the pond) has been maintained at a minimum distance of 200 m. These are conservative practices that have the effect of displacing the softer, weaker slimes farther inward and produce a thick, strong, dilative shell. In addition, three levels of internal drains have been installed in the shell as the dam has been raised (Fig. 6), in order to depress the phreatic surface; a fourth level will be constructed at elevation 175 m. As a result, the stability of the dam is not controlled by the tailings, but instead is controlled by the foundation soils (see Sections 6.2 and 6.3).

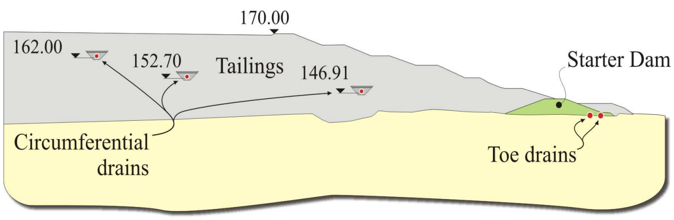


Figure 6. East Dam, location of the circumferential drains

The observational method has always been a major component of the design and operation of the Zelazny Most tailings dam. In order to monitor the deposition of the tailings and the performance of the dam (see Section 5), there are approximately 300 surface monuments for measuring horizontal and vertical movements. In addition, a total station with 23 micro-mirrors has recently been installed to more closely monitor one particular section of the dam. There are also approximately 1800 piezometers, of which 870 piezometers have their tips in the tailings, 830 in the foundation soils, and 100 in the starter dam. Finally, there are 42 deep inclinometers. In the future, the dam may also be monitored by means of satellite radar interferometry.

There are approximately 480 Mm^3 of tailings presently stored in Zelazny Most. The end of deposition is projected to occur in 2042 (an astonishing 65-year filling period: all of the authors of this paper will be in Geotechnical Heaven by that time), with a total volume of approximately 933 Mm^3. There are more than five disposal alternatives presently under consideration, including:

- Zelazny Most only: final crest elevation of 207.5 to 210 m asl;
- Zelazny Most plus a South-west expansion, possibly including some dry-stacking: final crest elevation of 195.0 m;
- Zelazny Most plus re-activation of an older tailings pond (named Gilów), with or without the South-west expansion: final crest elevation of Zelazny Most 180 to 185 m and Gilów 199 to 202 m;
- Zelazny Most only but drains the pond and continue with dry-stacking: final elevation 207.5 to 210 m;
- Zelazny Most plus paste thickening a portion of the tailings for disposal back into the underground mine workings plus either the Southwest expansion or Gilów: final crest elevation of 195 m.

The history of tailings disposal at Zelazny Most is roughly at the halfway point. Deposition (and then reclamation) will occur many years into the future. Different disposal technologies are under study (e.g. dry-stacking and paste thickening) in order to manage safely this enormous volume in an environmentally sound manner.

3. LOCAL GEOLOGY

The Zelazny Most tailings dam is situated on a complex sequence of geological deposits, the basic stratigraphy of which is summarised in Table 1. From the ground surface downwards these consist of Pleistocene deposits, which are lake clays and out-wash sands and gravels. These are underlain by Pliocene deposits, which include thick strata of plastic clays and also thin brown coals, below which are Triassic deposits (including beds of halite: sodium chloride), below which in turn lies the ore body. The presence of the halite is significant, since ground-water from the mines, which is used for processing the ore, is consequently saline. As a result, the decant water in the tailings lagoon is also saline, giving rise to potential environmental problems.

The complexity of these deposits, particularly of those immediately below the dam, results largely from the Pleistocene history of the area. During this period a succession of ice sheets moved from North to South over central Europe. At least eight major ice advances are recognised in Poland, the Southern limits of six of which occur in the country. At least three of these glacial limits lie South of Zelazny Most, though one of these only does so by few kilometres (Ber, 2006). Thus at three extensive ice sheets passed over Zelazny Most (Fig. 3), where at times the ice thickness probably exceeded a kilometre.

It is no surprise, therefore, that the upper part of the geological sequence has been significantly affected by glacio-tectonics induced by the over-riding ice, probably to depths of about 100 m. As a consequence, the Pliocene deposits, particularly, are intensely sheared, folded and generally disturbed. In places, the initially horizontally bedded freshwater Pliocene sediments (sands, silts and clays), now have Quaternary deposits thrust within them.

Where there are thicker layers of Pliocene clay the glacial thrusting has resulted in the formation of shear zones, presumed to have been moved in a North- South direction. These shear zones have major implications for the stability of the tailings dam; one of these has a North- South extent of at least 600 m within the Pliocene clay.

Table 1. Summary of geology and index properties

Age	Thickness	Lithology and index properties
Holocene	up to 4m	Alluvium. Variably clays, silty clays, sands, gravels. All these were removed from the dam foundations at construction.
Quaternary	up to 10m	Stratified glacial lake sediments: usually silty clay but sometimes clay.
	up to 30m	Fluvio-glacial sediments: usually sand (occasionally gravelly), often thinly (20cm) interbedded with silt or clay.
	up to 50m	Moraine clays: variable silty, more usually sandy clays, which become more sandy or gravelly with depth. All the Quaternary clays have properties in the ranges: w_{nat} = 7 - 26%; w_L = 20 - 50%; w_P= 10 - 18%; I_P = 10 - 30%; I_L = 0.02 - 0.22.
Pliocene	100 - 150m	Clays, silty clays, silty sands, sands; rarely coarse sand and/or gravel. w_{nat} = 9-26%, w_L = 50-80%, w_P = 15-25%, I_P = 30-50%, but more plastic under North and East Dams, with w_L sometimes >90%, w_P >60%.
Trias		
Ore body		

3.1 Solifluction

As a consequence of the cold climate towards the end of the Pleistocene, after the last ice sheet to have reached Zelazny Most had melted, the phenomenon of solifluction is likely to have occurred. This process, which involves shearing at shallow depth probably due to mud slides, results in the presence of polished shear surfaces at or close to residual strength (e.g. Skempton 1964). As with the glacio-tectonic shear zones, but on a much smaller scale, these also have implications for the stability of the dam where they occur.

Experience in the UK, particularly in areas South of the major Pleistocene glacial limit, shows that solifluction shear surfaces occur extensively at shallow depths in clay soils. Where the slopes exceed about 7-8°, the mudslides were fairly extensive, and the shear surfaces run approximately parallel to the existing ground surface. Where the ground surface is less than 7-8°, down to inclinations as small as 2°, the solifluction shear surfaces are discontinuous, where the mud slides were tiny. Solifluction shears are usually encountered at relatively shallow depths, usually not much greater than 4-5m, often less. A classic example of a failure involving the presence of discontinuous solifluction shear surfaces is the Carsington Dam (Skempton 1985; Skempton and Coates 1985; Skempton and Vaughan 1993), which collapsed during construction when the dam had reached a height of about 40 m.

Detailed investigations at Zelazny Most, where the natural ground slopes rarely exceed 4-5°, showed that, as in the UK, there was solifluncted clay with discontinuous shear surfaces.

Three categories of solifluction were reported: low; medium; and high intensity; and also ‘clay with/without solifluction structures’.

However, the solifluction shear surfaces did not occur below depths of 1.5 m, and were rather infrequent, and moreover were rarely inclined in a direction critical for the stability of the dam.

Consequently, for the medium and high intensity solifluncted clay it was assumed that strength parameters $c_r'=0$, $\varphi'=\tan-1(1.4 \times \tan \varphi_r')$ were assumed to apply to a depth 1.5 m below the original ground surface. The multiplying factor of 1.4 allows for the discontinuous nature of the shear surfaces in the solifluncted clays. These parameters are used for design where clay soil occurs in the dam foundation just below the original ground surface.

4. GEOTECHNICAL SITE CHARACTERIZATION

4.1 Foundation Soils

Characterisation of the foundation soils is needed for several reasons. These include: (1), to establish the foundation geology, including its index properties; (2), to determine strength properties for limit equilibrium stability analyses; and (3), to estimate the in-situ stress state, stress-strain behaviour, and undrained and effective stress strength parameters for finite element analysis. This has been done by sampling from downstream of the toe of the dam (the "fore-field") so as far as possible to be sure that the soil is unaffected by the construction of the dam.

In doing this for the foundation soils it was quickly recognised that for stability the Pliocene clays were the most important element of the foundation, and so it is the characterisation of the clay that for this purpose is of greatest concern.

There are very few precedents for the construction of a major structure on stiff plastic clay which has been subject to glacio-tectonics. Perhaps the closest analogy that has been reported in the literature is Empingham Dam, in the United Kingdom, which is founded on the Upper Lias clay at a location where it has been subjected to the periglacial phenomenon of cambering (Kovacevic et al. 2007; Chandler 2010). Both at Zelazny Most, and at Empingham, the dam foundations have been affected by a stress field that is presumed to approximate to simple shear, and in both cases the consequential strains are substantial, probably well in excess of 100%. In these situations where the ground has been pre-sheared, the properties of the clay are very different from those expected of a conventional overconsolidated clay, which is usually presumed to have had a much simpler stress history, affected only by vertical unloading due to erosion.

Once the clay has been sheared, either by glacio-tectonics or by cambering, it is likely to be much less brittle than conventional stiff clay. Moreover it is to be expected that there will be extensive shear zones along which the strength has been reduced to, or close to, residual strength. The in-situ stresses will have changed, and may be close to passive if the ground surface is more or less horizontal (as at Zelazny Most), or less than passive if the ground is sloping and is unable to sustain the full magnitude of the passive stresses (as at Empingham).

Where clays exist below the dam foundation any existing shear zones will have an important influence on stability, and the in-situ stresses and undrained strength of the clay will also be important. The effect of the glacio-tectonics has resulted in extremely variable soil conditions, and it is to be expected that the in-situ stresses and undrained strength of the clay will be similarly variable.

These properties have all been examined with an extensive programme of undisturbed sampling and laboratory testing so as to provide information on the degree of variability of the physical properties of the foundation soils, particularly the clay soils which are critical for the stability of the dam, and hence allow a rational choice of the relevant parameters for design purposes, both for limit equilibrium and finite element analyses.

4.1.2 Estimating the in-situ Stress State in the Pliocene Clay.

The suction in samples of the Tertiary clay from the fore-field of the dam was measured so as to assess the in-situ stress state, knowledge of which is required for finite element analysis of the performance and deformation of the dam. This characterisation must be affected by the geological history of the clay, which is complex. First, erosion, including that resulting from the passage of Pleistocene ice, will have induced considerable over-consolidation, and second the glacio-tectonics are to be expected to have deformed the clay extensively, resulting in the clay being in a different state from that expected if it had only been subjected to K_0 conditions during normal consolidation and subsequent vertical unloading by surface erosion.

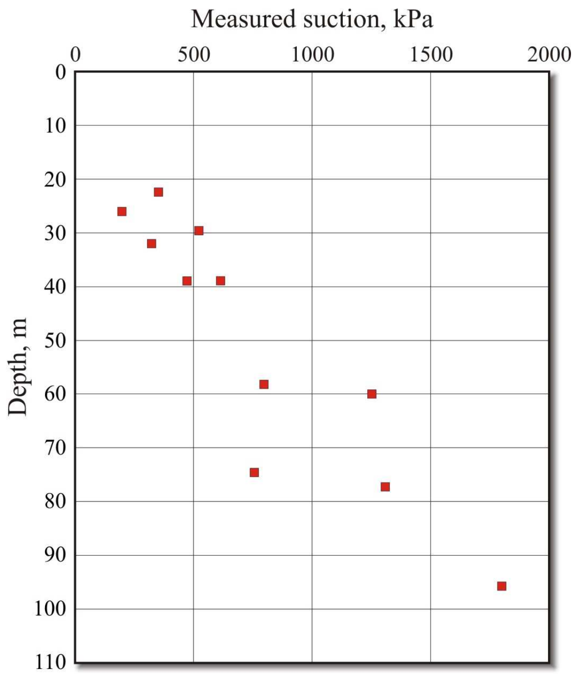
All samples were taken using a double-tube rotary sampler; they were of high quality. They were extruded shortly after extraction from the ground, trimmed to remove any disturbed material that might have caused a change in the suction of the sample due to pore water migration towards the centre of the sample, and then wrapped in aluminium foil to prevent evaporation. The samples were then either transferred to the testing laboratories for the triaxial testing, or were used for suction measurements at the on-site laboratory. This latter testing was carried out as soon as practical after sampling, sometimes within an hour, using the Ridley/Burland suction probe (Ridley and Burland, 1993), following the procedure they describe.

If it is assumed that the samples behave perfectly elastically when extracted from the ground, then the measured suction is related to the coefficient of earth pressure by the equation

$$p_k = \sigma_v'[K - A_S(K - 1)] \qquad (1)$$

where p_k is the measured suction, σ_v' is the vertical effective stress, and A_S the pore-pressure coefficient (Skempton 1954) due to sampling, which is taken as ⅓, and K is the coefficient of earth pressure, σ_h'/σ_v'. Since it is anticipated that the ice sheets will have applied a stress state akin to simple shear, the earth pressure coefficient K is unlikely to be K_0.

The values of p_k measured are given in Fig. 7 and the corresponding computed values of K are reported in Fig. 8.



N.B.: Samples with significant amounts of silt have been removed from this plot.

Figure 7. Suction measured in Pliocene clay, Ridley (2008)

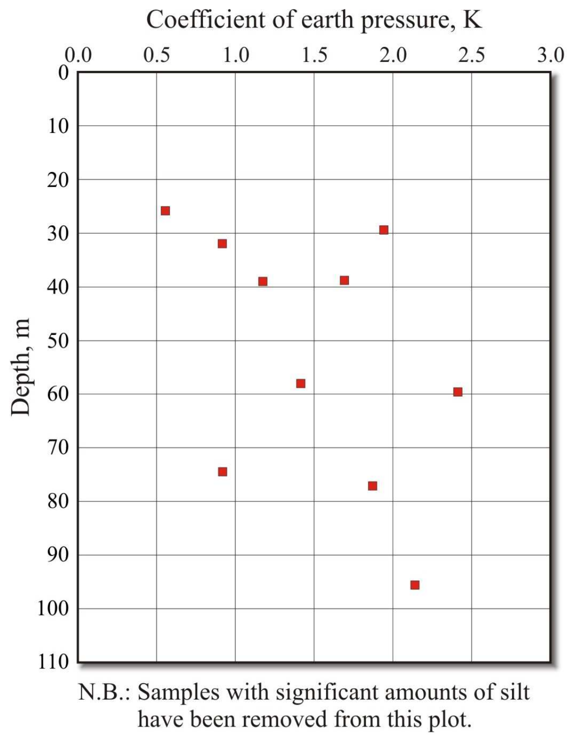


Figure 8. Coefficient of earth pressure of Pliocene clay

4.1.3 Bulk Strength

Stress-strain behaviour. The stress strain behaviour of the Zelazny Most clays is typically non-brittle, much less brittle than conventional overconsolidated clays. Typical stress - strain curves are shown in Fig. 9.

Unconsolidated undrained (UU) strength. Another major design decision required for both limit equilibrium and finite element analyses is the value of bulk strength in the foundation. Given the possible presence of extensive clay lenses at shallow depth, and the presence of thick clay strata at greater depth, it is the properties of the clay that are of most concern, particularly the likelihood of undrained behaviour. The latter is clearly an issue since high construction pore pressures are being observed.

The likely bulk strength of the clay is being examined using rotary cored double-tube core barrels, the drilling using polymer flush. Unconsolidated undrained strengths are measured after the surface of the sample has been trimmed so as to minimise loss of effective stress due to post-sampling swelling.

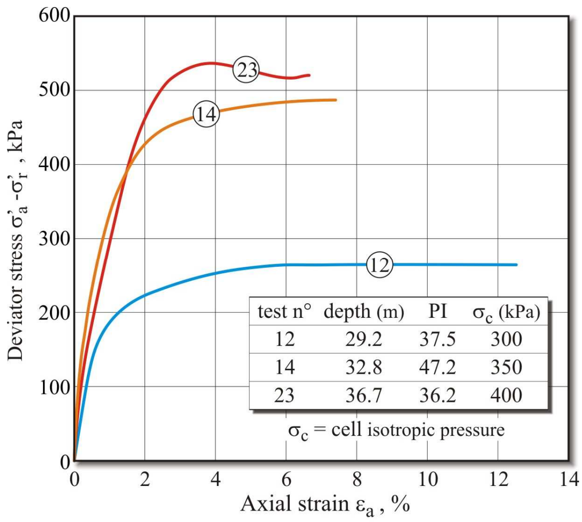


Figure 9. TX-UU tests on Pliocene clay, examples of stress- strain curves.

Initial effective stress is measured in the triaxial apparatus, and the compressive strength then measured using slow undrained shearing and measurement of pore pressure.

This procedure allows measurement of both undrained strength and effective stress strength, though only the undrained shear strength, $S_U = (\sigma_1 - \sigma_3)/2$, is discussed here. The undrained strength versus depth are not shown here, but, as expected for such a glacially disturbed clay, there is considerable scatter in the strength data, though the strength shows an increase with depth.

Residual Strength. The presence of major shear zones in the clay in the foundations of the dam requires knowledge of the residual strength. This was measured using the Imperial College/Norwegian Geotechnical Institute ring shear apparatus (Bishop et al., 1971), the testing being carried out under drained conditions on remoulded samples of clay having a range of plasticity indices (IP), under vertical stresses in the range 150 to 1,000 kPa. See example shown in Fig. 10.

The strength parameters chosen for design, however, are only indirectly related to those obtained from the ring-shear data. There are two main reasons for this.

1. In the two locations beneath the dam where clays occur in significant extent the direction of the Pleistocene ice movement (which was approximately North– South) is either at about 90° or 180° to the likely movement of the dam, so that any re-activation of movement will not be in the direction of formation of the glacio-tectonic shear zones.

2. It has been known for some time that the strength mobilised when existing landslides are re-activated is typically $\varphi_r'+2°$, where φ_r' is measured in the ring shear apparatus. This assumes that c_r' is taken as zero. The reason for this is that the true failure surface in a landslide is almost always considerably more irregular than is assumed in the back-analysis, leading to a higher mobilised strength.

As a consequence of these two factors it has been assumed that the strength on the observed shear zones is $c'=0$, $\varphi'=10°$, as discussed later. It is anticipated that as further data become available, particularly that relating to mechanisms of potential failure, these strength parameters may need revision.

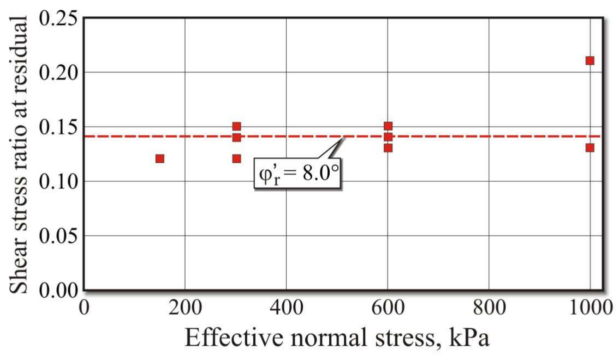
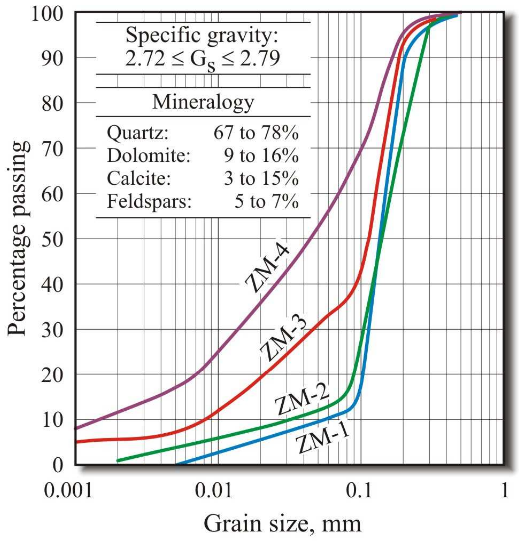


Figure 10. Ring shear residual strength

4.2 Tailings

Considerable work has been carried out over the last twenty years in an attempt to achieve a comprehensive and reliable geotechnical characterisation of the tailings. The results of this work are summarised here.

Coarse tailing from the Rudna and Lubin Mines are used to raise the dam, whilst the much finer tailings from the Polkowice Mine are deposited hydraulically directly in the pond. The range of particle size distribution of the Rudna and Lubin tailings were investigated by Lipinski (2000; 2005), and these are shown in Fig. 11 together with their mineralogical composition and the range of specific gravity, G_s. The values of IP of the more plastic fine tailings do not exceed 12; many samples are non-plastic.



		ZM-1	ZM-2	ZM-3	ZM-4
D_{50}	mm	0.17	0.12	0.10	0.041
Sand	%	88	86	66	42
Fines	%	12	14	34	58

Figure 11. Tailings grain size distribution

Because of the difficulties in sampling the tailings in their undisturbed state, characterisation was mainly attempted using a variety of in-situ tests. Hundreds of static cone penetration tests, both without (CPT) and with pore pressure measurements (CPTU), have been carried out. Examples of these tests, showing the cone resistance (q_c) measured during CPTU tests carried out on the beach of the East Dam, at various distance from the dam crest, are shown in Fig. 12.

When the inherent variability of the q_c values with depth resulting from spigotting is taken into account, the cone resistance of the tailings is seen generally to decrease as the distance from the dam crest increases. This can be ascribed to the increase in fine content as the test location gets closer to the pond. Similar comments can be made regarding the results of two seismic Marchetti's flat dilatometer [S-DMT] tests (Marchetti et al. (2008; 2009) run on the beach of the East Dam at distances of 40 and 200 metres from the dam crest, as shown in Fig. 13.

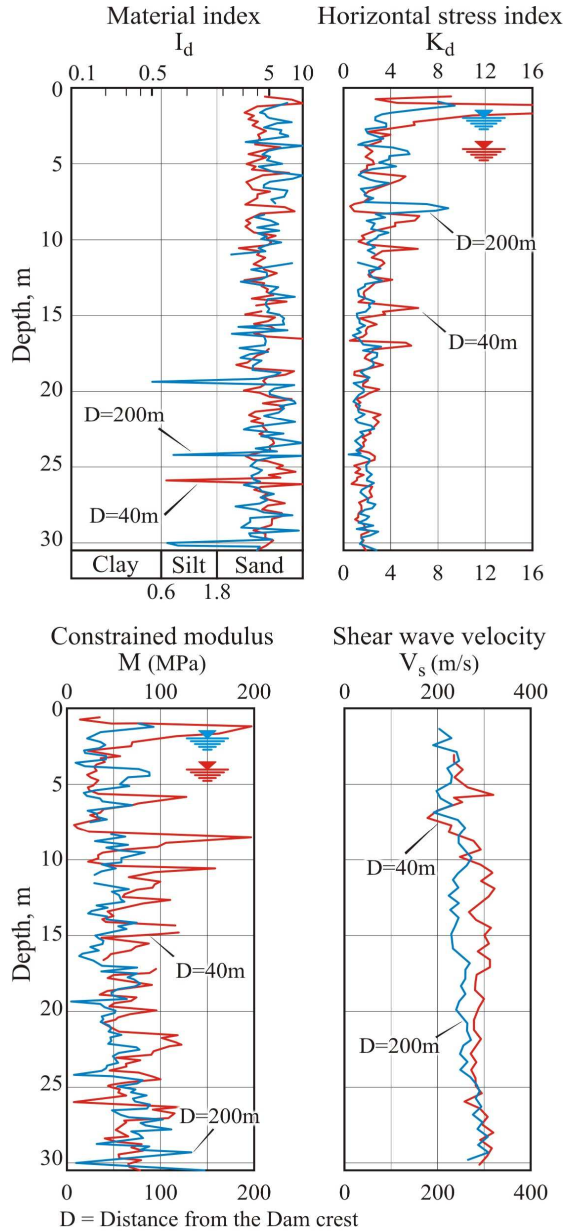


Figure 13. East Dam, results of two S-DMT's

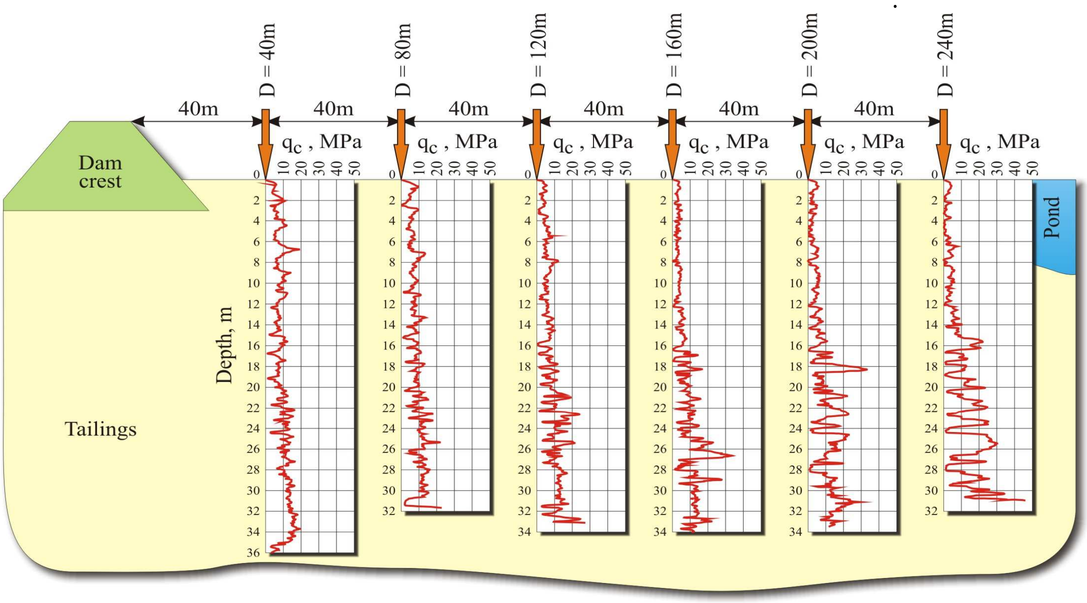


Figure 12. West Dam, cone resistance at different distances from the dam crest

Table 2. West Dam, Standard Penetration Tests results

D=40m FC=30%			
z (m)	N b/ft	ER %	$(N_1)_{60}$ b/ft
9.3	16	82	17
11.2	32	85	32
14.2	52	86	46
16.6	30	79	22
18.1	33	80	24

D=120m FC=32%			
z (m)	N b/ft	ER %	$(N_1)_{60}$ b/ft
9.6	20	76	20
11.3	4 ?	80	9
12.7	23	89	23
16.0	13	85	18
18.0	13	86	11

D=200m FC=52%			
z (m)	N b/ft	ER %	$(N_1)_{60}$ b/ft
9.8	2	91	3
11.0	13	90	15
13.6	20	86	24
15.5	31	86	35
16.7	28	80	28

N = uncorrected b/ft
$(N_1)_{60}$ = corrected b/ft
ER = energy ratio
FC = fines content

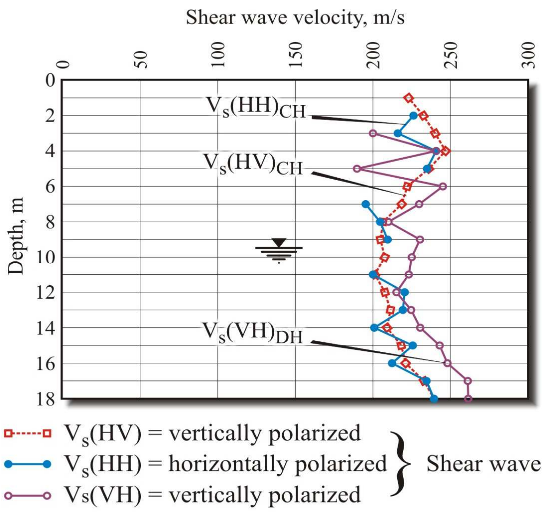


Figure 14. West Dam (40 m from the crest), shear wave velocity from cross-hole and down-hole tests

Table 2 provides details of some Standard Penetration Tests (SPT) result performed in bore-holes located at the West Dam beach, at three different distances from the dam crest. During the tests the energy delivered to the driving rod was measured, allowing the blow-count to be normalised to that of 60% of the nominal test energy, and with respect to an effective overburden stress (σ'_{V0}) of 100 kPa, leading to the values of $(N_1)_{60}$ shown in the table.

Particular attempts were made to characterise the mechanical behaviour of the tailings using geophysical methods. Several cross-hole (CH) and down-hole (DH) tests were carried out to measure seismic compression (V_p) and the shear wave (V_s) velocities. An example comparing the two types of test is shown in Fig. 14. In addition, a number of seismic static cone penetration tests (S-CPTU) and the two mentioned seismic dilatometer tests (S-DMT's) were carried out, during which V_s was measured. In all seismic tests the wave velocity was measured using two receivers, allowing the assessment of V_p and V_s by the true time interval method.

In addition, some tests were carried out using the electrical resistivity cone penetrometer (E-CPTU), in which the soil resistivity was measured in both the horizontal and vertical directions, as well as measurement of the resistivity of the pore fluid itself.

For reasons of space, only the results of seismic wave velocity measurements within the tailings are discussed here, concentrating on the following issues.

- Small strain shear modulus G_o = f (V_s), and constrained modulus M_0 =f (V_p); this latter relationship is only applicable to partially saturated tailings.
- Evaluation of the in-situ void ratio (e_0) as function of V_p and V_s; see Foti et al. (2002) who proposed the formula

$$n = \frac{\rho_s - \left[\rho_s^2 - \dfrac{4(\rho_s-\rho_f)K_f}{V_p^2 - 2\left(\dfrac{1-\nu_s}{1-2\nu_s}\right)V_s^2}\right]^{0.5}}{4(\rho_s-\rho_f)} \tag{2}$$

which holds only for fully saturated soils.

ρ_s =soil particles } mass; K_f =bulk modulus of pore fluid
ρ_f =pore fluid } density; ν_s =Poisson ratio of soil skeleton

- Assessment of the saturation line and the spatial distribution of fully saturated tailings based on the measured V_p values.
- Quantitative assessment of the susceptibility of the saturated (or nearly saturated) tailings to static liquefaction, based on the value of V_s.

Examples of cross-hole tests performed at different distances from the dam crest on the West Dam are seen in Fig. 15.

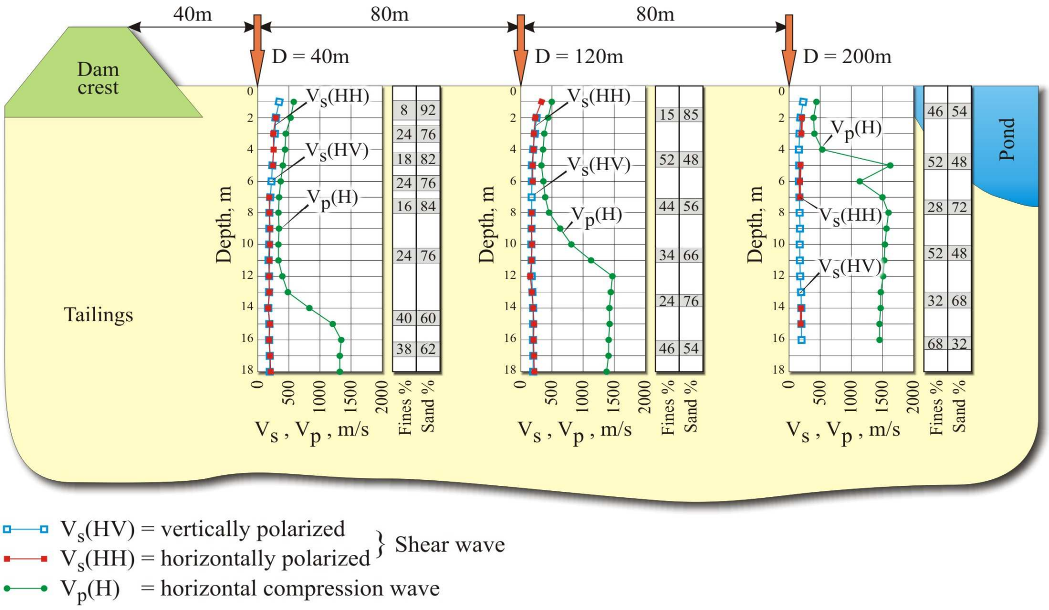


Figure 15. West Dam, cross-hole tests results

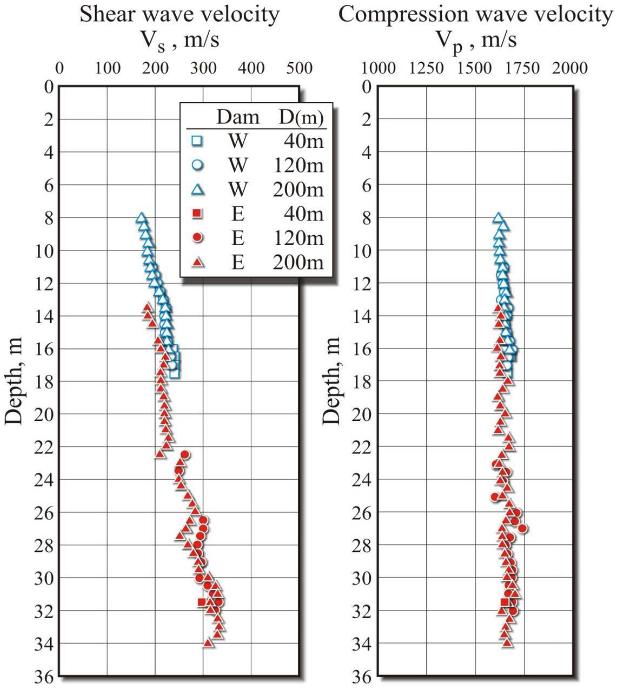


Figure 16. East Dam, seismic wave velocity of saturated tailings

These show the measured values of the velocities V_p and V_s, the latter obtained using waves polarised in both vertical V_s (VH) and horizontal V_s (HH) planes.These test data show that the values of V_s are generally quite high for normally consolidated sand-silt deposits, varying between 150 and 250 m/s, with the usual trend of increasing values with depth as result of increasing effective overburden stress.

The values of V_s (VH) and V_s (HH) suggest (Fig. 15) that the tailings have very limited initial (i.e. small strain) anisotropy.

The V_p values confirm their usefulness as a sensitive indicator of the depth at which the tailings become fully saturated; see Valle Molina (2006). Assuming that saturation coincides with the depth at which the V_p values exceeds 1500 m/s, on the cross-section considered the saturation line can be identified as occurring at depths of 14.5m (D = 40m), 11.0m (D = 120m) and 7.0m (D = 200m), D being the distance from the dam crest.

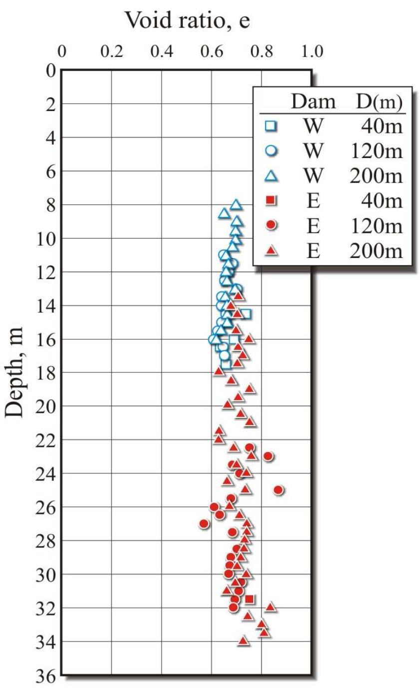


Figure 17. East Dam, void ratio from CH tests results

Once the position of the saturation line has been identified, the values of e_o can be estimated using the formula of Foti et al. (2002). These calculations were carried out for a series of nine cross-hole tests carried out in 1993 (3 tests), and in 1997 (6 tests). Fig. 16 shows the values of V_s and V_p of saturated tailings used in the calculation of eo for the six tests performed in 1997, while Fig. 17 shows the values of e_o computed from the cross-hole test results.

With the exception of classification and index property tests, laboratory tests are largely limited to consolidated undrained triaxial tests. These were carried out both in compression and extension, on isotropically and anisotropically consolidated specimens (TX-CIU; TX-CAU tests), aimed at investigating the susceptibility of the tailings to flow failure, or static liquefaction (Casagrande 1976, Castro 1969, Ishihara 1993). The majority of these triaxial tests were performed on specimens reconstituted using the 'under-compaction' method (Ladd, 1974) with a few further tests using pluvial deposition and slurry sedimentation (Lipinski 2000). All the reconstituted samples, except for a few prepared by slurry sedimentation, exhibited a pronounced susceptibility to flow failure, as seen in Fig. 18 (see next page), yielding an undrained residual strength, s_{ur} lower than 0.2 times the current effective overburden stress.

This behaviour of the reconstituted samples resulted in the serious concern that there was a risk that a flow failure of the tailings might trigger a collapse of the ring-dams. Consequently, in 1993, it was decided to attempt "undisturbed" sampling of the tailings in hand-dug pits located in the beach at varying distances from the dam crest. Thin-wall samplers, 70mm in diameter and 140mm in height, were pushed in at the bottom of the trial pits, at depths of 2 to 3 m. The samples were sent to the soil mechanics laboratories of Warsaw Agriculture University, the Norwegian Geotechnical Institute, Oslo, and the ISMGEO in Bergamo, Italy, and, after saturation, TX-CIU and TX-CAU tests were carried out. Although the adopted sampling method does not guarantee entirely undisturbed specimens, the tests showed a very different behaviour of tailings during undrained shearing from that obtained in the earlier series of tests. All the undisturbed specimens initially exhibited contractive behaviour, passing through the point of phase transformation (Ishihara 1993) and then exhibiting continuous dilation as shown in Fig. 18.

Up to the present time, three series of these tests on "undisturbed" specimens have been carried out; Hoeg et al. (2000), Dyvik (1998), and Lipinski (2005). All the tests have yielded similar results, confirming the different response of "undisturbed" compared to reconstituted specimens. Table 3 shows the results of the series of tests carried out in 1998, comparing the behaviour of undisturbed and reconstituted specimens. This table also shows the values of the initial shear modulus, G_o , deduced from the value of V_s measured in triaxial tests using bender elements. It is seen that not only is the strength of the "undisturbed" samples greater, but that the values of G_o are also constantly higher. It seems that specimens taken from the trial pits in the beach, despite the non-ideal sampling procedure, have preserved, at least partially, the in-situ soil fabric of the tailings.

Table 3. Undrained triaxial tests results on saturated tailings, Dyvik, 1998

L (m)	FC (%)	e_0 (-)	Test* type	B-range (-)	$(G_0)_1$ (bar)	$\frac{(G_0)_1 U}{(G_0)_1 R}$
40	22	0.811	U	0.968 to 0.991	849	1.23
		0.809	R	0.976 to 0.996	691	
120	24	0.806	U	0.950 to 0.987	851	1.20
		0.821	R	0.971 to 0.994	709	
200	28	0.810	U	0.967 to 0.995	857	1.27
		0.808	R	0.980 to 0.995	674	

(*) TX-CAU-CL, σ'_{rc} ; 50, 250 and 500 kPa, $\frac{\sigma'_{rc}}{\sigma'_{ac}} = 0.5$

L = Distance from the dam crest
$(G_0)_1 = G_0$ at $\sigma'_m = 1$ bar
FC = Fine content, low plasticity silt
U = Undisturbed specimens
R = Reconstituted specimens

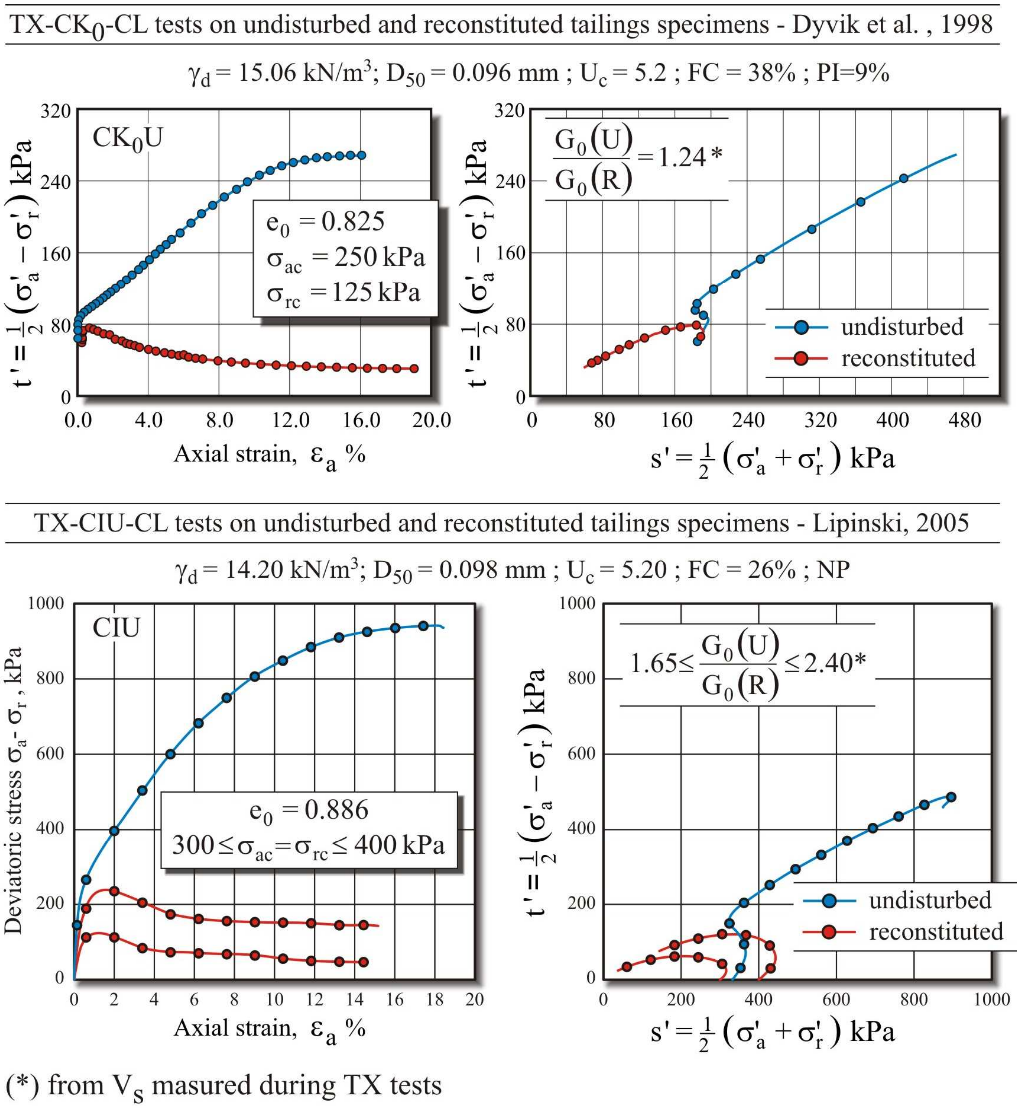


Figure 18. Examples of undrained triaxial compression tests on tailings

5. MONITORING

The design and incremental construction of the ring dam around the Zelazny Most reservoir relies on the use of the observational method. The dam is raised on average 1.25 m per year by the upstream construction method, and the observations are compared with predicted performance and form the basis for any adjustments required in the design to safely proceed to higher elevations. The field instrumentation and monitoring system, including data presentation, quality control and evaluation, has been steadily improved over the years and especially since about the year 2000.

The field instrumentation includes piezometers installed in all segments of the ring dam (North, East, South and West dams) and in the foundation under the dam. Surface displacements on the dam and downstream of the dam toe are measured at 350 benchmarks, including the heads of the 50 inclinometers that have been installed. GPS as well as geodetic surveying is used to measure surface displacements. In 2008 a modern total station was installed close to the East Dam to ensure reliable and frequent measurements of surface displacements for this segment, which exhibits the largest displacements. The reference benchmarks installed for the total station have also increased the accuracy of the conventional geodetic surveying.

During the last 8 years several inclinometers have been installed down to depths greater than 100 m after it was realized that significant shear displacements took place much deeper in the foundation than first anticipated (see Section 6.3). Vibrating-wire piezometers were installed in the same borings as the inclinometers, using the fully grouted method of installation for piezometers (Vaughan, 1969); Mikkelsen and Green, 2003; Countreraas et al. 2007). Seepage weirs have been installed to measure the seepage/leakage coming from the circumferential strip drains installed at different levels in the dam, as well as from the toe drains of the starter dam.

Mining-induced seismicity and the effects of vibrations imposed on the dam are of concern (see Section 6.1 below). Therefore, accelerometers are installed in some cross-sections of the West Dam which is closest to the mining activity.

6. SELECTED GEOTECHNICAL PROBLEMS

6.1. Mining Induced Seismicity

The natural seismicity is relatively low in the region of the Zelazny Most tailings dam. Of main concern is the seismicity induced by the nearby underground mining. The copper mines utilize a form of room-and-pillar extraction process, in which the void is backfilled and most of the pillars are removed.

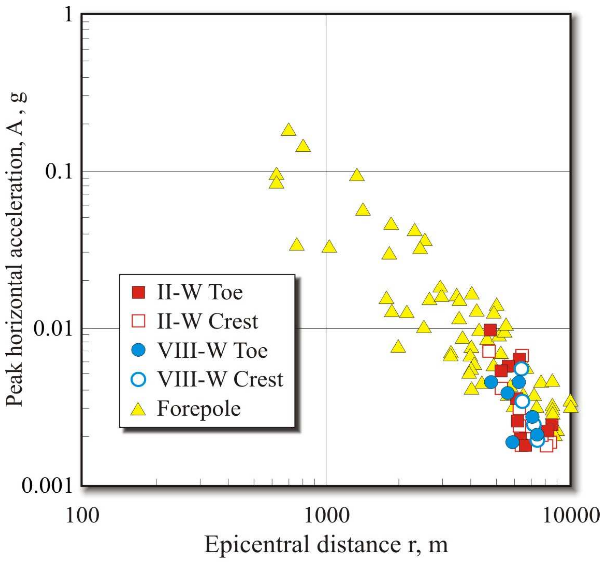


Figure 19. Peak ground acceleration vs. epicentral distance

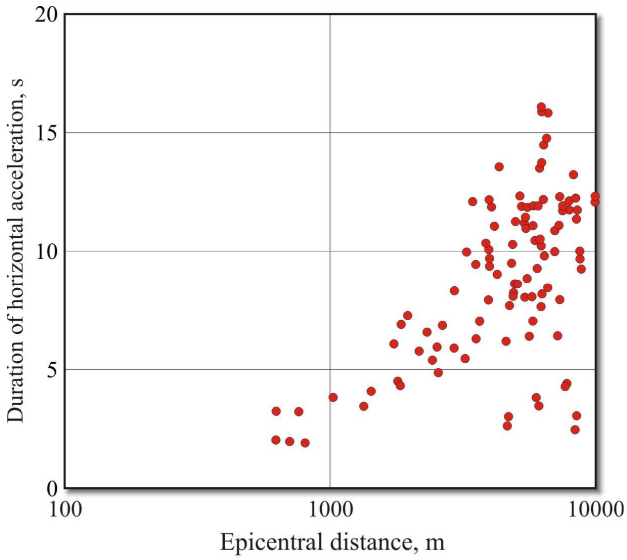


Figure 20. Duration of shaking vs. epicentral distance

These pillars spall due to load re-distribution, and produce tremors. An array of seismometers and accelerometers has been established on the land surface above the mines, and at selected points on the dam. Thousands of tremors have been detected and recorded. For most of these events, the seismic energy has been calculated and the epicentral location has been estimated, including depth.

Experience has shown that only the events with seismic energy greater than $1x10^7$ joules are of concern; and then only if they occur near the dam. In recent years, such events have occurred roughly once-a-month: From the end of 2007 to the end of 2008, 13 events occurred. For these 13 events, a total of 104 records were obtained from various instruments, including four located at the toe and crest of the dam on the west side. (All mining is occurring west of the dam; no mining is planned North, South, or east.) Fig. 19 presents peak horizontal ground acceleration (PGA, at frequencies less than or equal to 10 Hz) versus epicentral distance for these 13 events.

For purposes of design and monitoring of the dam, pseudo-static slope stability analyses are performed using a horizontal seismic coefficient equal to one-half of the design PGA. Because of the large size of the dam, the design value of the PGA varies from 0.040 g on the east side (farthest from the mining) to 0.100 g on the west side. As shown in Fig. 19 the maximum peak horizontal acceleration experienced at the dam during 2008 was 0.010 g, on the west side, at an epicentral distance of 4719 m.

Thus, during this period, the west side of the dam experienced approximately 10% of the design PGA. However, a few years ago, it experienced approximately 29% of the design PGA. Furthermore, as the epicentral distance decreases, the acceleration increases. As shown in Fig. 19, the maximum acceleration that was measured in 2008 in the western foreground of the dam was 0.183 g, at an epicentral distance of 707 m from the instrument. This was the same event that produced 0.010 g at the dam.

There is a 750-m "no mine" zone beyond the toe of the dam, but these recent seismic data indicate that the design values of the PGA could be exceeded as the underground mining approaches closer and closer. In response, two actions have been taken: (1) The miners have altered their operations, promising that no events greater than $1x10^7$ J will occur in the vicinity of the dam; (2) Dynamic finite element analyses are being performed to supplement and complement the conservative pseudo-static slope stability analyses (see Section 6.3). As is well-known, pseudo-static assumes a constant horizontal force, whereas, the seismic data in Fig. 20 indicate the duration of shaking is only a few seconds for small epicentral distances, corresponding to high PGA: compare with Fig. 19 (Duration is defined approximately as the period over which the measured acceleration exceeds 5% of the PGA).

As shown in Fig. 20, the duration generally increases as the epicentral distance increases and the corresponding PGA decreases. Thus, it is anticipated that the dynamic finite element analyses will confirm the safety of the dam at high PGA, but short duration.

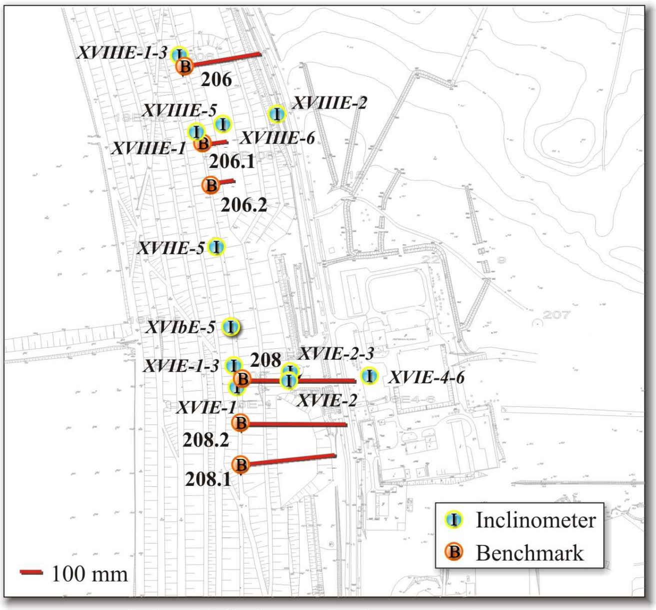
The land surface above the mines has settled approximately 1 m due to the extraction process. Because of the depth of the mines, the settlement is uniform and gradual. However, a tremor occurred a few years ago that caused coffee to slosh out of cups in the hotel in the town of Polkowice, which is located approximately 7600 m west of the dam. Some of the guests even ran out of the hotel.

6.2. Flow Failure of Tailings

In view of the substantial number of flow (or liquefaction) failures that have occurred of tailings dams, e.g. Morgenstern (1998), Blight (2009), the susceptibility of the Zelazny Most tailings to static liquefaction was, and continues to be, one of the major engineering concerns given the continued raising of the confining dams. The theoretical framework of the mechanism of flow failure in sandy-silty soils was established by Casagrande and his co-workers (Casagrande 1976, Castro 1969). Further insights into this problem are provided by Ishihara (1993) and Yamamuro et al. (1999).

The susceptibility of the Zelazny Most tailings to static liquefaction is regularly monitored as the construction of the dams proceeds, using both in-situ and laboratory tests. In-situ tests are primarily the cone resistance q_c measured by the CPTU's (Fig.13), and V_s measured during CH and DH tests (Figs.15 and 16), while triaxial tests have been, and continue to be carried out in the laboratory on specimens retrieved from trial pits excavated in the beach, as described in Section 4.2. The triaxial test results, illustrated in Fig. 9, show that the samples preserve their in-situ fabric, at least partially, and do not appear to be prone to flow failure.

The values of V_s and q_c analysed in light of Robertson et al. (1992) and Stark and Olson (2003), also suggest that the risk of the flow failure is low. In particular, the measured V_{s1} values, normalised with respect to the vertical effective stress, generally result in values greater than the value of 160 m/s which is regarded by Robertson et al. (1992) as the upper bound shear velocity above which the tailings dilate in shear. It is worth mentioning that this criterion, based on the value of V_{s1}, leads to similar conclusions to those provided by Stark and Olson (2003), who reported q_c values for a number of case records of flow failures of existing tailings dams.



Horizontal displacements of eastern starter dam since benchmarks installation in early eighties.

Horizontal displacements of eastern starter dam - Period: Sept. 2008 - Sept. 2009

Benchmark:	208-2	208	208-1	206-2	206	206-1
Displacement, mm:	37	43	40	28	20	26
Installed year:	2003	1981	2003	2003	1981	2003

Inclinometer:	16E-1	16E1-3	16bE-5	17E-5	18E-1	18E-05	18E1-3
Displacement, mm:	43	41	38	28	19	17	20
Installed year:	2004	2004	2005	2005	2001	1997	2004

Figure 21. East Dam, vectors of horizontal displacements

6.3. Measured Dam and Foundation Displacements

6.3.1 Displacements for Different Segments of the Ring Dam

Fig. 5 shows a plan view of the approximately 15 km long ring-dam. The locations of the dam cross-sections referred to below are indicated on the figure. The East Dam crosses the old river bed and constitutes the highest segment of the dam. At present (2010), cross-section XVIE is 60 m high, and this is the cross-section that exhibits the largest horizontal and vertical displacements. Fig. 21 shows the recorded horizontal displacement vectors for different points around the ring dam. The rate of horizontal displacements (mm/yr) is the highest at cross-section XVIE; however, since 2006 fairly high horizontal displacement rates have also been recorded for the North Dam. A more detailed account of the development of displacements of the East and North dams is presented below and points out concerns about the increasing rate of horizontal displacements.

6.3.2 Recorded Horizontal Displacements for the East Dam

Table 4 shows a comparison of the average rates of horizontal displacements recorded in cross-sections XVIE and XVIIIE at the crest of the starter dam (135 m asl) during different time periods since 1981.

Table 4. Rates of maximum horizontal displacements recorded during different time periods for cross-section XVI-E and XVIII-E.

Period	1981-1994	1995-1997	1998-2001	2002-2007	2008-2009
XVIII-E, Point 206	102 mm 8 mm/year	60 mm 20 mm/year	105 mm 26 mm/year	123 mm 22 mm/year	23 mm/year
XVI-E, Point 208	92 mm 7 mm/year	74 mm 24.6 mm/year	221 mm 44.2 mm/year	237 mm 40 mm/year	45 mm/year

There was a significant increase in the horizontal displacement rates during the period 1995-1997, when the crest level reached about elevation 155 m asl.

The crest of the dam in 2010 is at 172 m asl. The average rates have remained fairly constant since 1998, about 40 – 45 mm/yr for cross-section XVIE and about 22 mm/yr for cross-section XVIIIE. Fig. 22 shows a plot of the recorded horizontal and vertical displacements vs. time since 1981 of cross-section XVIE. Experience with the recorded displacements has shown that, due to measurement inaccuracy prior to the installation of the total station, it may be misleading to draw conclusions about rates of displacements based on geodetic measurements over time periods shorter than 1 year. However, recent measurements seem to show that there is a slightly higher rate of horizontal displacements during the summer than the winter period. Since 1981 the crest of the starter dam at cross-section XVIE has moved in the downstream direction a total of 570 mm (Fig. 22).

During the measurement period August 2005–August 2006, the maximum rate of horizontal surface displacement in cross-section XVIE was reported to be as high as 50 mm/year, and the heave at the toe of the starter dam to be 15 mm/year. This caused concern, and in September 2006 the crest of a segment of the East Dam (spigotting sections E0 - E3) was relocated towards the centre of the pond (maximum relocation distance was 165 m) to decrease the future average downstream slope. During the subsequent year, September 2006–September 2007, the rate of horizontal displacement in cross-section XVIE was reported to have decreased to about 25 mm/year, and virtually no vertical heave was recorded at the toe of the starter dam. For cross-section XVIIIE, the horizontal displacement rate was also reported to have decreased significantly compared to that recorded during the previous year, and was about one half of that in cross-section XVIE.

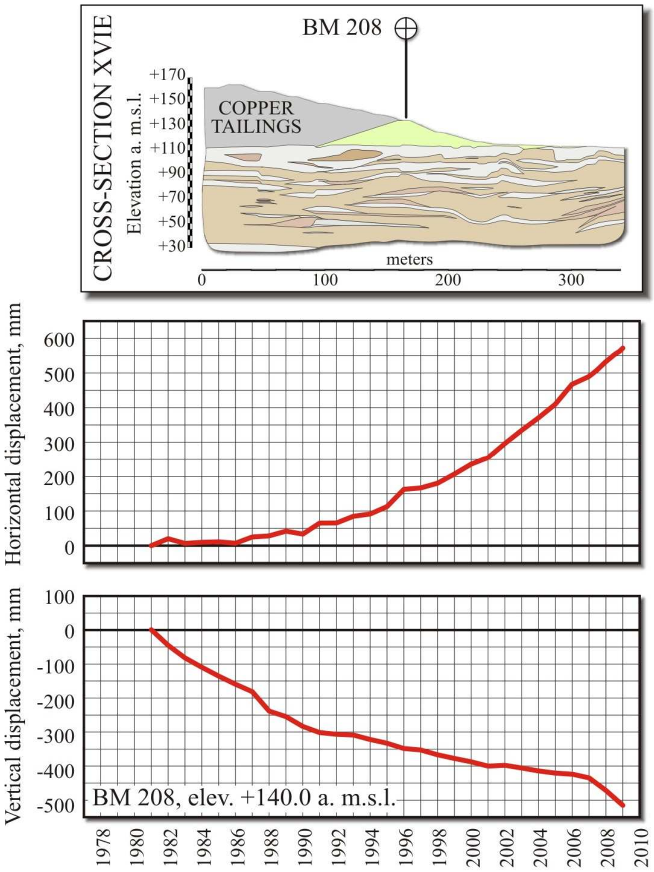


Figure 22. East Dam, horizontal and vertical displacements of the benchmark 208 (crest of the starter dam).

However, for the next measurement period, September 2007–September 2008, the maximum horizontal displacement rate in cross-section XVIE was 44 mm/year, which is almost the same as that measured prior to the crest relocation. In cross-section XVIIIE the maximum horizontal displacement rate during September 2007–September 2008 was recorded to be 23 mm/year, which is almost the same as that measured during the period August 2005– August 2006. The high displacement rates continued to occur even though the crest level across the spigotting sections E0 – E3 had not been raised during the previous 1½ years (i.e. no load had been added). It is unlikely that consolidation in the dam and its foundation has any significant effect on horizontal displacements. Therefore, the continuing rate of displacements must primarily be caused by shear-creep deformations. As the measured rates of horizontal displacements in cross-sections XVIE and XVIIIE were almost the same as before the dam crest was relocated, it was concluded that for this segment of the ring dam the relocation has had very little beneficial effect on the rate of horizontal displacements. In retrospect, one may suspect that the recorded temporary reduction in horizontal displacement rate was a measurement error or an error in interpretation.

During the subsequent year, September 2008–September 2009, the maximum horizontal displacement recorded in cross-section XVIE was 47 mm, and the maximum vertical settlement was 53 mm at the same point. The horizontal displacement is very similar to that observed during the previous year. During the period September 2008–September 2009 the relocated crest was raised ca. 2 m.

The geodetic survey data clearly shows the pronounced "settlement trough" at cross-section XVIE and about 100 m to each side of that cross-section (Fig.23). Cross-section XVIIIE, some 350 m North of cross-section XVIE, is almost outside the trough and shows much smaller settlements and present rates of settlements. The trough has become much more pronounced in recent years, especially during the year September 2007–September 2008 when the settlement rate was about 40 mm/year compared to about 15 mm/year during the two previous years. During the period September 2008–September 2009 the settlement rate was even higher, about 50 mm/yr. The regional surface settlement due to the influence of mining exploitation is reported to be 2 - 6 mm/year, and cannot explain this significant increase in the rate of vertical settlement. However, the rate has undoubtedly been accelerated by the pumping from the drainage wells, which began in April 2008, and the start of the construction of the stabilizing toe berm as described in Section 6.4.2.

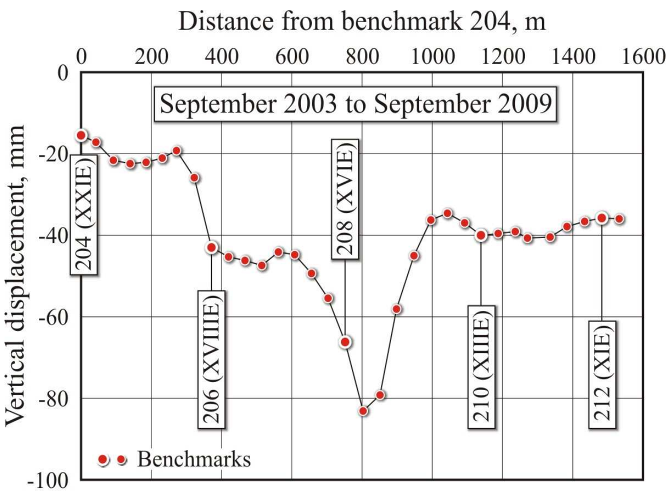


Figure 23. East Dam, vertical displacement along the crest of the starter dam

6.3.3 Recorded Horizontal Displacements for the North Dam

The rate of horizontal displacement in cross-section XVIN during the period September 2006 –September 2007 increased significantly compared to the rate during the previous year. The maximum horizontal displacement reported during the period September 2006–September 2007 was about 20 mm/year, and there was also some toe heave. The magnitude and rate of these displacements caused concern, because cross-section XVIN is little more than half the height of cross-section XVIE, and also parts of the North dam rest on Tertiary plastic clay (see Chapter 3). For the period September 2007–September 2008 the maximum rate of horizontal displacements in cross-section XVIN is reported to be the same as for the previous year, and for the period September 2008–September 2009 the rate was 23 mm/yr with some tendency to toe heave. Thus, the rate has stayed constant for the last 3 years at about 20 – 23 mm/year.

For the period September 2007–September 2008 the maximum rate of horizontal displacement in cross-section VaN was the same as that during the previous year, about 25 mm/year, and during the period September 2008 – 2009 the same horizontal displacement was recorded with a toe heave of 8 mm.

Unexpectedly, during the year September 2008–September 2009, a very large maximum horizontal displacement was recorded in cross-section IXN (between sections VaN and XVIN) of as much as 38 mm and a toe heave of 9 mm. As part of the new inclinometer installation programme, an inclinometer is currently being installed in this section to determine if the significantly increased horizontal displacement rate is the result of a zone of concentrated shear strain in the Pliocene foundation clay, similar to that observed under the East Dam (see Section 6.3.4).

Since the start of the available displacement record in 1994 for the North Dam, the maximum accumulated horizontal displacements in cross-sections XVIN and IXN are about 170 mm, while those in cross-section VaN are only slightly smaller, about 150 mm. The horizontal displacement rates in sections XVIN and VaN have about doubled during the last 3 years, while the rate during the last year increased even more in cross-section IXN.

6.3.4 Results of Inclinometer Readings for the East Dam

As the tailings dam is being built by the upstream construction method, the main concern about instability was initially regarding the tailings dam itself and not the foundation. Therefore, the early inclinometers installed through the dam and into the foundation of the East Dam were fairly shallow. They turned out to be too shallow to detect the development of significant shear displacements that later proved to take place at large depths. The deep inclinometers installed after 2003 clearly define movements on a sub-horizontal shear zone in the foundation of cross-section XVIE at about 40 m asl, i.e. some 75 m beneath the original ground surface. Fig. 24 shows the recorded inclinometer measurements in that cross-section.

An interpretation of the results from this and the neighbouring inclinometers in sections North (up to cross-section XVIIIE) and South (down to cross-section XIVE) of cross-section XVIE indicates that a shear zone at 40 – 50 m asl extends at least 600 m along the dam axis, and at least 150 m from the toe under the dam. There is as yet (2010) no indication that movement at this horizon is occurring downstream of the dam toe or under the higher, upstream portion of the dam. However, the extent of the concentrated sub-horizontal shear zone under the dam is not yet fully defined, and additional inclinometers are currently being installed to define better the zone of concentrated shear strain.

During the years 2003-2007 the measured horizontal displacement rates at 40 m asl recorded by the inclinometers were about the same as the maximum measured horizontal dam displacement recorded on the crest of the starter dam at cross-section XVIE.

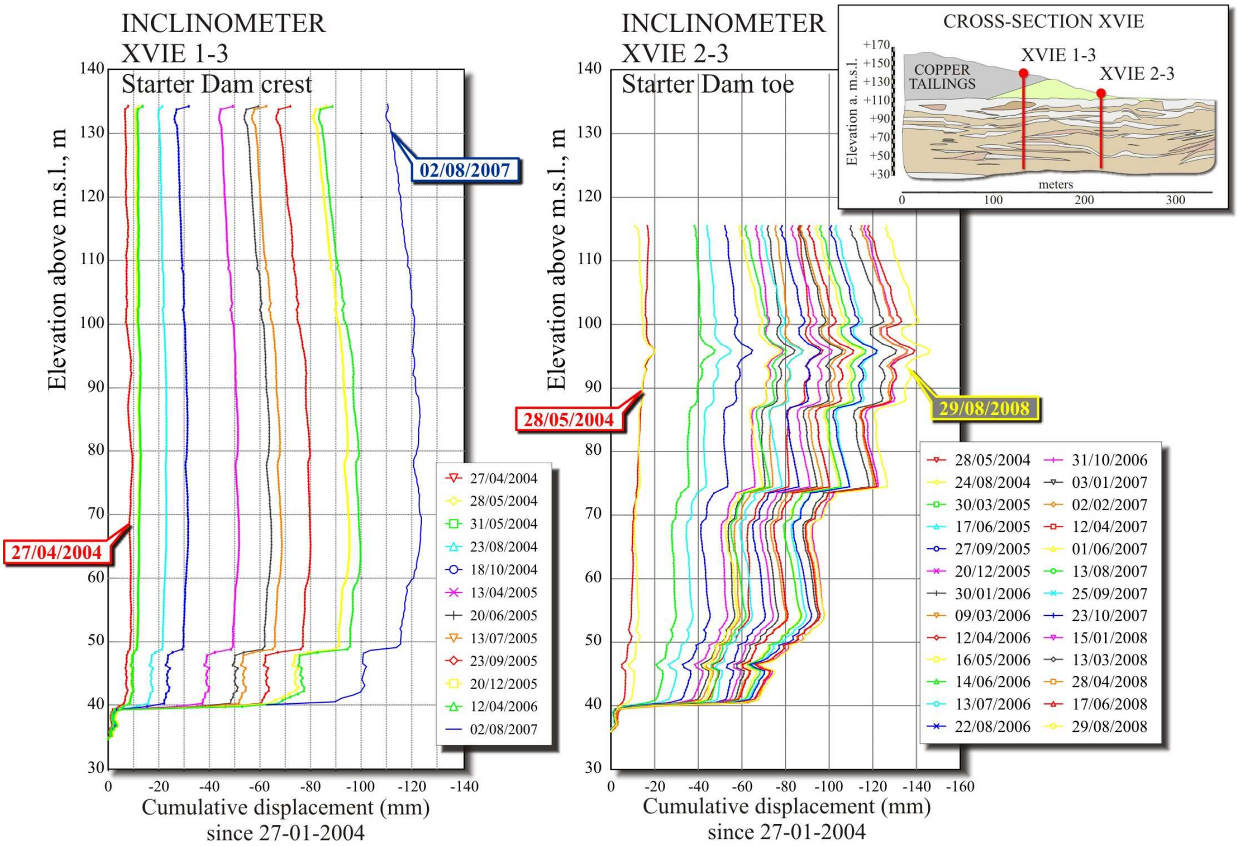


Figure 24. East Dam, horizontal displacements measured in deep inclinometers

Therefore, by combining the information from the incremental surface displacement vectors, the inclinometer readings, and the recorded movements of the gallery under cross-section XVIE, it was concluded that the foundation mass above 40 m asl, together with the dam, was moving as a near rigid body, sliding along the shear surface in the Pliocene clay at 40 m asl. However, since about 2007, the inclinometer displacement at that elevation has gradually become less than the horizontal surface displacement. The difference is due to additional shear distortions above 40 m asl, especially at 75 m asl where a plane of concentrated shear seems to develop, some 40 m beneath the ground surface.

6.4. Stability Analyses for East Dam

6.4.1 Potential Failure Mechanism and Computed Safety Factors

Extensive limiting equilibrium analyses have been performed searching for the critical failure surfaces for the different dam and foundation cross-sections. Using the location of the zone of concentrated shear defined by the inclinometers, the critical failure body for cross-section XVIE was determined to be as shown in Fig. 25.

The shear strength parameters used for the tailings and the foundation materials are defined in Chapter 4. The pore pressures used for the stability analyses are those that were actually observed. They are consistent with steady-state seepage in the tailings dam and in the pervious strata of the foundation. However, in the clay strata of the foundation there are significant excess construction pore pressures as measured by the vibrating-wire piezometers.

In the concentrated shear zone in the Tertiary clay, of presumed glaciotectonic origin, the strength parameters are taken as $c' = 0$, $\varphi' = 10°$, which assumes that the mobilised value of φ' would be about 2° above φ_r' determined by the ring shear tests, as discussed in Section 4.1.3.

In the sequence of interbedded lenses and layers of clay, silt and sand in the foundation from above the glaciotectonic shear zone up to the original ground surface, uniform parameters of $c' = 0$ and $\varphi' = 25°$ are used. The use of $c' = 0$ was considered appropriate since the material has been highly disturbed by the over-riding Pleistocene ice.

b

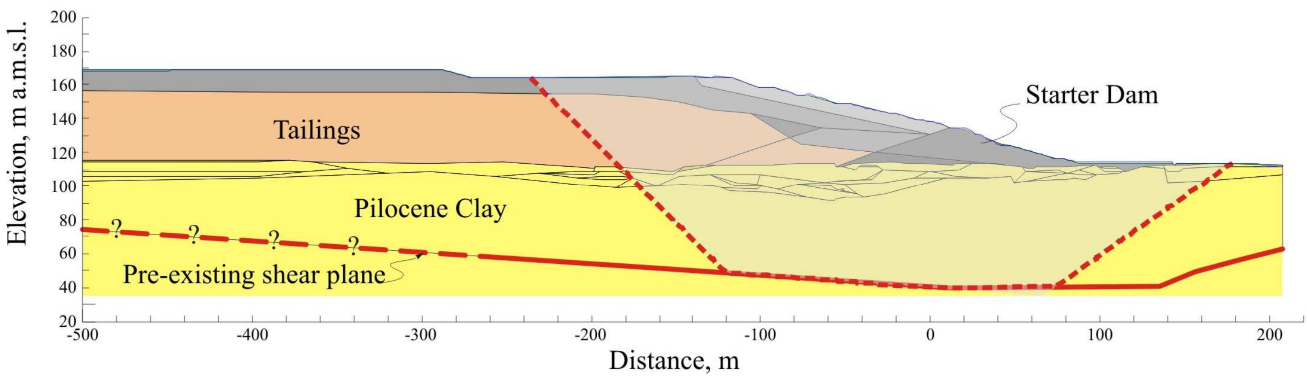


Figure 25. East Dam, cross-section XVI-E, critical failure body

With these input parameters, the computed factor of safety was found to be 1.62. If the angle of shearing resistance in the interbedded lenses and layers of clay, silt and sand from above the glaciotectonic shear zone up to the original ground surface is conservatively reduced from 25° to 20°, the computed factor of safety is 1.40 (i.e. a reduction of 15%). If, in addition, the angle of shearing resistance along the glaciotectonic shear zone is reduced from 10° to 7°, the computed factor of safety is 1.27 (i.e. a further reduction of 9%).

As shown by the inclinometer under the starter dam, and described in Section 6.3.4, there is indication of another shear zone at 75 m asl. If a limiting equilibrium analysis is performed with the sliding taking place at this level, and using the same shear strength parameters as used to compute $F = 1.62$ above, the computed factor of safety is 1.43. Consequently, if the upper and lower shear zones had the same strength parameters, it would be expected that the horizontal movement would be at 75 m asl, and not as observed at 40 m asl. Possible explanations that this does not occur are that the upper shear zone has higher shear strength than the lower shear zone, and/or that the upper zone is less extensive.

Considering the relatively high rate of observed horizontal displacement for cross-section XVIE (45 mm/year), one would have expected somewhat lower computed factors of safety. Therefore, to gain a better appreciation of the situation than a limiting equilibrium analysis by itself can give, a finite element stress-strain-time analysis is currently being undertaken to shed light on the deformations in the dam and its foundation and the local shear strength mobilization.

All the stability analyses described above were performed without taking into account the effects of mining-induced vibrations. Pseudo-static analyses have been performed using a seismic coefficient equal to ½ of the assumed peak horizontal ground acceleration/g at different locations around the ring dam. The West Dam experiences the highest accelerations, while the East Dam far away from the mining operations, is assumed to experience peak ground accelerations no higher than 0.04g. Even with a seismic coefficient = 0.02, the reduction in static safety factor for the failure body is about 7%. However, it is realized that the use of a pseudo-static analysis for the mining-induced vibrations with very high frequency and very short duration is not realistic. Therefore, a more appropriate dynamic analysis is currently underway to estimate the permanent deformations that might occur due to the assumed maximum mining-induced vibrations.

6.4.2 Measures to Improve Stability

The present crest elevation of the Zelazny Most tailings dam is 172 m asl. The immediate goal is to raise the dam to 180 m asl by the year 2016, and subsequently up to 205 m, if possible. Therefore, various ways of improving stability have been and are being studied:

- Construction of stabilizing toe berms at critical cross-sections
- Move dam crest inward towards the pond to flatten the average downstream slope
- Reduce foundation pore pressures by drainage measures. (The phreatic line and pore pressures inside the tailings dam itself have been lowered by constructing the circumferential strip drains.)

For cross-section XVIE with a very deep potential sliding surface through the foundation, a stabilizing berm at the toe has to be very large to give a significant effect. However, at this location there is not much space downstream of the toe, unless a public road is relocated and several buildings, including a pumping station, are removed. Presently a berm is being constructed, but without, at this time, having to relocate any of the existing facilities. However, according to the limit equilibrium computations, the factor of safety will only increase by a few %, and the berm will have to be made much larger to provide an increase of 10-20%.

By moving the dam crest inwards towards the pond, the reduction in the rate of horizontal displacements for cross-section XVIE was insignificant. However, it is not known what the displacement rate might have been if the crest had been raised without the relocation. This is the subject of ongoing numerical stress-strain-time analyses.

The East Dam rests on a very deep foundation with glaciotectonic shear zones in layers of plastic Pliocene clay. Other segments of the dam do not rest on such foundations, and here the relocation of the dam crest towards the pond may have significant stabilizing effects. Therefore, this option for stabilizing the ring dam should not be discarded. However, the relocation of the crest will, of course, reduce the storage capacity of the tailings deposit.

Various schemes for achieving some foundation drainage and reduction of pore pressures on potential failure surfaces have been considered. This has included the drilling of sub-horizontal drains under the dam from the dam toe and applying under-pressures. Excavating drainage galleries under critical sections of the dam and drilling drains from such galleries have also been contemplated. However, the geology of the dam foundation is extremely complex and erratic, and it was considered questionable whether such expensive undertakings could be justified considering the uncertain outcome of the undertaking.

It is a much less difficult task to drill vertical drainage wells and to apply continuous pumping to the wells. This has been done on a trial basis, and the effects have been evaluated. Five wells were installed close to cross-section XVIE, and screened over a significant depth from approximately 100 m asl to 30 m asl. One well was located at the toe of the dam (115 m asl) and the other four at the crest of the starter dam (135 m asl) approximately 25 m apart. The effect of the pumping is relatively small away from the wells, but the reduced pore pressures have been included in the limiting equilibrium stability analyses. The factor of safety was found to be increased by 3%. The beneficial effect was considered significant enough for 7 more wells to be installed in the area of cross-section XVIE to further study the reduction in foundation pore pressures and the increase in computed factor of safety.

7. CLOSING REMARKS

The paper describes the geotechnical problems and the investigations involved in the design and construction of Europe's largest copper tailings disposal facility, at Zelazny Most in South-west Poland. Despite the extreme complexity of the engineering problems, the following general points may be made.

- A thorough study of the local geology is essential for the selection of a site for a large tailings disposal facility. That this study was not completed in sufficient detail during the siting of the facility is by far the most important problem for the safety of the tailings disposal at Zelazny Most.
- The paper summarises the work that has been carried out to characterise satisfactorily both the foundation soils and the tailings. This involves the choice of the most appropriate in-situ and laboratory testing methods so as to establish a detailed characterisation of the stress-strain-strength behaviour of the geomaterials involved.
- Given the extended time period over which the construction and operation of the tailings disposal works has developed, and will continue to develop in the future, the use of the observational method, using an extensive and redundant monitoring system, appears to be the only way to maintain and control the geotechnical safety of the dam.

8. ACKNOWLEDGEMENTS

The writers express their thankfulness to the Management of KGHM for consenting the publication of this paper.

Moreover it is deeply acknowledged the significant help provided by Mr K. Janicki (MSc) and Mr. J. Lewinski (MSc) for their valuable support in collecting the soil investigation and the monitoring data presented in the paper.

9. REFERENCES

Ber, A., 2006. Glaciotectonic Map of Poland. Państwowy Instytut Geologiczny, Warsaw.

Bishop, A.W., Green, G.E., Garga, V.K., Andresen, A. and Brown, J.D., 1971. A new ring shear apparatus and its application to the measurement of residual strength. Géotechnique 21, issue 4, 273-328.

Blight, G., 2009. Geotechnical Engineering for mine waste storage facilities. CRC Press, Taylor & Francis Group.

Carrier, W.D., 2003. Stability of tailing dams. XIX Conference of Geotechnics of Turin on slope stability and stabilization, Turin, Italy.

Casagrande, A., 1975. Liquefaction and cyclic deformation of sands: a critical review. V Panamerican Conference on SMFE, Buenos Aires, Brazil, vol 5, 80-133.

Castro, G., 1969, Liquefaction of sands. Ph.D. Thesis, Harvard soil mechanics series no. 81. Harvard University, Cambridge Mass. USA

Chandler, R.J., 2010. Stiff sedimentary clays: geological origins and engineering properties. Géotechnique 60, in press.

Contreraas,I.A. Grosser, A.T. and Ver Strate, R.H., 2007. The use of the fully-grouted method for piezometer installation. VII International Symposium on Field Measurements in Geomechanics, ASCE GSP 175, Boston MA, USA.

Dyvik, R. and Hoeg, K., 1999. Comparison of tests on undisturbed and reconstituted silt and silty sand, *in* Physics and mechanics of soil liquefaction. International Workshop Lade, P.V. and Yamamuro, J.A., Eds, Baltimor, Maryland, USA. Balkema, 59-167.

Dyvik, R., 1998. Triaxial tests of undisturbed and reconstituted specimens of tailings. Report of the Norwegian Geotechnical Institute, Oslo.

Foti, S., Lai, C.G.and Lancellotta, R., 2002. Porosity of fluid-saturated porous media from measured seismic wave velocities. Geotechnique 52, issue 5, 359-373.

Hoeg,K., Dyvik, R. and Sandbaekken, G., 2000. Strength of Undisturbed versus Reconstituted Silt and Silty Sand Specimens, ASCE Journal of Geotechnical and Environmental Engineering, Vol.126, No.7, 606-617.

ICOLD, 2001. Tailings Dams: Risk of dangerous occurrences: lessons learnt from practical experiences. Bulletin 121, International Commission on Large Dams, Paris (jointly published with United Nations Environment Programme).

Ishihara , K., 1993. Liquefaction and flow failure during earthquakes. Geotechnique 53, issue 3, 351-415.

Kovacevic, N., Higgins, K.G., Potts, D.M. and Vaughan, P.R., 2007. Undrained behaviour of brecciated upper Lias clay at Empingham dam. *Geotechnique* 57, issue 2, 181-196.

Ladd, R.S., 1974. Specimen preparation and liquefaction of sands. Technical Note. ASCE J of Geotechnical Engineering Division, 100, issue 10, 1180-1184.

Ladd, R.S., 1978. Preparing test specimens using undercompaction. ASTM Geotechnical Testing Journal 1, issue 1, 16-23.

Lipinski, M. J., Wolski, W., Fioravante, V., Jamiolkowski, M.B. 1997. Preliminary evaluation of hazard due to liquefaction for Zelazny Most tailings pond. XIV ICSMFE, Hamburg, vol. **3**, 1843-1846.

Lipinski, M.J., 2000. Undrained response of cohesionless soils to monotonic loading. Ph.D. thesis, Technical University of Gdansk, Poland.

Lipinski M.J., 2005. Evaluation of change in shear strength of Żelazny Most tailings material. Technical Report, Geoteko Ltd.

Marchetti, S., Monaco, P., Totani, G. and Marchetti, D., 2008. In situ tests by seismic dilatometer (SDMT). Volume honouring J.H. Schmertmann, ASCE, GSP 180, 292-311.

Mikkelsen, P.E. and Green, G.E., 2003. Piezometers in fully grouted boreholes. VI International Symposium on Field Measurements in Geomechanics, Oslo, Norway, 545-554

Monaco, P. and Marchetti, S., 2007. Evaluating Liquefaction Potential by Seismic Dilatometer (SDMT) accounting for Aging. IV International Conf. on Earthquake Geotech. Engng., Thessaloniki, Paper 1626.

Morgenstern, N.R. 1998. Geotechnics and Mine Waste Management - Keynote Lecture. International Symposium on Seismic and Environmental Aspects of Dam Design, Santiago, Chile.

Ridley, A.M. and Burland, J.B., 1993. A new instrument for the measurement of soil moisture suction. Geotechnique 43, issue 2, 321-324.

Skempton, A.W., 1954. The pore pressure coefficients A and B. Geotechnique 4, issue 4, 143-147.

Skempton, A.W., 1985. Geotechnical aspects of the Carsington Dam failure. XI ICSMFE, San Francisco, vol 5, 2581-2591.

Skempton, A.W. and Coats, D.J., 1985. Carsington dam failure. in *Failures in Earthworks, ICE Symposium* London, Thomas Telford, 203-220.

Skempton, A.W. and Vaughan, P.R., 1993. The failure of Carsington dam. *Geotechnique* 43, issue 1, 151-174.

Valle Molina, 2006. Measurements of V_p and V_s in dry, unsaturated and saturated sand specimens with piezoelectric transducers. PhD thesis Texas University at Austin, Texas.

Vick, S.G., 1983. Planning, design and analysis of tailings dams. John Wiley & Sons Inc., New York.

Vaughan, P.R., 1969. A note on sealing piezometers in boreholes. Geotechnique 19, issue 3, 405-413.

Yamamuro J.A., Covert, K.M. and Lade, P.V., 1999. Static and cyclic liquefaction of silty sands. In Physics and mechanics of soil liquefaction. International Workshop, Lade, P.V. and Yamamuro, J.A., Eds, Baltimor, Maryland, USA. Balkema, 55-66.